DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND SOUTHWEST
INTEGRATED PRODUCT TEAM WEST
2001 JUNIPERO SERRA BOULEVARD, SUITE 600
DALY CITY, CALIFORNIA 94014-1976

IN REPLY REFER TO

Ser 05/485
April 14, 2005

Mr. Phillip A. Ramsey
U.S. Environmental Protection Agency
Region IX
75 Hawthorne Street
San Francisco, CA 94105

Re: RESULTS OF LIQUEFACTION STUDY AND LANDFILL GAS STUDY, SITE 1, TIDAL AREA LANDFILL, NAVAL WEAPONS STATION SEAL BEACH, DETACHMENT CONCORD, CONCORD, CALIFORNIA

Dear Mr. Ramsey,

1. In response to comments received on our December 24, 2004 "Pre-Final (95%) Remedial Design, Landfill Cover, Tidal Area Landfill, Site 1, Naval Weapons Station Seal Beach, Detachment Concord" (pre-final design), the Navy had an evaluation performed of the potential for liquefaction at the subsurface at Site 1 during an earthquake. In a letter dated January 20, 2005 the Navy provided you with a copy of the work plan for this study. Today, we are pleased to provide the U.S. Environmental Protection Agency (U.S. EPA) with the results of this study in the enclosed report titled "Liquefaction Study, Site 1 Tidal Area Landfill, Naval Weapons Station Seal Beach, Detachment Concord." As planned, the results of this study will be incorporated into the draft final version of the "Closure Plan and Post-Closure Maintenance Plan," which will be submitted with the final design in compliance with State applicable or relevant and appropriate requirements (ARARs) specified in the Site 1 Record of Decision (ROD) for the landfill cover. A primary conclusion of the study is that the cover will not require any special design features to accommodate earthquake-induced movements.

2. The Navy is also pleased to present the U.S. EPA with the enclosed report titled "Landfill Gas Characterization, Site 1 Tidal Area Landfill, Naval Weapons Station Seal Beach, Detachment Concord." The landfill gas study was conducted as promised in the ROD to ensure that the design adequately considers any landfill gas generation. The results of this study indicate that there is no significant landfill gas generation.

3. As neither of the enclosed reports are Primary or Secondary documents under the Federal Facility Agreement (FFA), the Navy is providing them as informational and not seeking

comments. However, if you do have any concerns or questions regarding them, please do not hesitate to contact me at telephone No. 650-746-7451 or Internet e-mail: stephen.f.tyahla@navy.mil.

Sincerely,

Stephen F. Tyahla, P.E., CHMM
Lead Remedial Project Manager

Enclosures
Copy to:
California Department of Toxic Substances Control Region 1 (Attn: Jim Pinasco)
California Regional Water Quality Control Board, SFBAY (Attn: Laurent Meillier)
Contra Costa County Environmental Health, LEA (Attn: Agnes T. Vinluan)
Cal/EPA Integrated Waste Management Board Permitting & Enforcement Division (Attn: Frank Davies)
California Department of Fish and Game (Attn: Frank Gray)
Restoration Advisory Board (RAB) Co-Chair (Attn: Ms. Mary Lou Williams)
RAB Member Lisa Anich
RAB Member Kevin Cornish
RAB Member David Griffith
RAB Member Gregory Glaser
RAB Member Jessica Hamburger
RAB Member Ed McGee
RAB Member Mario Menesini
RAB Member Ray O'Brien
RAB Member Igor Skaredoff
Clearwater Consultants (Attn: Patrick Lynch)
Tech Law, Inc. (Attn: Jennifer Hollingsworth)
NWS Seal Beach, N45WS (Attn: Margaret Wallerstein)
NWS Seal Beach, N09WS (Attn: Gregg Smith)
EFD Southwest (3) (Diane Silva- Admin Record/IR/Base copy)
IPT West (Attn: Lik-See Chung)
TtEMI San Francisco (Attn: Joanna Canepa)
TtEMI San Francisco (Attn: John Bosche)

GENERAL SERVICES ADMINISTRATION
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Liquefaction Study
Site 1 Tidal Area Landfill

Naval Weapons Station Seal Beach Detachment Concord
Concord, California

GSA.0032.0017

April 15, 2005

Department of the Navy
Integrated Product Team, West
Daly City, California

TETRA TECH, INC.

GENERAL SERVICES ADMINISTRATION
Contract Number: GS-10F-0076K
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GSA.0032.0017

Liquefaction Study
Site 1 Tidal Area Landfill
Naval Weapons Station Seal Beach Detachment Concord
Concord, California

April 15, 2005

Prepared for

Naval Facilities Engineering Command

DEPARTMENT OF THE NAVY
Steve Tyahla, Engineer-in-Charge
Integrated Product Team West
Daly City, California 94014-1976

Prepared by

TETRA TECH EM INC.
135 Main Street, Suite 1800
San Francisco, California 94105
(415) 543-4880

John Bosche, P.E., Project Manager

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ACRONYMS AND ABBREVIATIONS

ASTM	American Society for Testing and Materials (now ASTM International)
bgs	Below ground surface
CCR	California Code of Regulations
cm^2	Square centimeter
CPT	Cone penetrometer test
CSR	Cyclic stress ratio
DMG	California Division of Mines and Geology
DO	Delivery Order
FS	Feasibility study
g	Gravity
$M \geq 6.7$	Greater than or equal to magnitude 6.7
M6.5	Moment magnitude earthquake of 6.5
mm	Millimeter
MCE	Maximum credible earthquake
MPE	Maximum probable earthquake
NCEER	National Center for Earthquake Engineering Research
PGA	Peak ground acceleration
RI	Remedial investigation
SBT	Soil behavior type
Site 1 Landfill	Tidal Area Landfill
SPT	Standard penetration test
Tetra Tech	Tetra Tech EM Inc.
USGS	U.S. Geological Survey
WG02	Working Group on California Earthquake Probabilities, 2002
WG99	Working Group on California Earthquake Probabilities, 1999

1.0 INTRODUCTION

The Naval Weapons Station Seal Beach Detachment Concord (NWSSBD Concord) Tidal Area Landfill (Site 1 Landfill) is being remediated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) process. Figure 1 shows the location of the Site 1 Landfill.

A final record of decision (ROD) was completed under CERCLA for the cover at the Site 1 Landfill. The ROD identifies the substantive closure standards for the remedial design (RD), which is the next phase of the CERCLA process. The RD requires development of design documents that contain the elements of a closure plan as described in Title 27 *California Code of Regulations* (CCR) Sections 21769 and 21830. An element of the closure plan is this liquefaction study, which evaluates liquefaction potential at the site. In addition, the study evaluates the magnitude of landfill settlement and lateral movement in the event of soil liquefaction during earthquake shaking. This liquefaction study complies with the requirements of CCR Division 2, Title 27 (Title 27 CCR).

Loose, granular material tends to compact and become denser when it is shaken. When such material is below the groundwater or is otherwise saturated, compaction causes water pressure to increase in the spaces, or pores, between the grains. Pore water pressure can build up excessively during an earthquake, which can cause a decrease in effective stress and a corresponding reduction in the shear strength of the soil. Effective stress is the difference between the weight of the soil above and the pore water pressure at that depth. Shear strength is the resistance of the soil grains to shearing, or movement relative to each other.

When soil liquefaction occurs, the decrease in effective stress and reduction in shear strength can result in movement of the liquefied soil layer. In cases where soil liquefaction occurs, there can be lateral movement and settlement of the ground surface. A general concern for landfills is that lateral movement or settlement associated with soil liquefaction under the landfill might disturb or damage the landfill cover. Potential settlement or lateral movement of the landfill resulting from soil liquefaction during earthquakes should be considered in the design of the cover. Various design and construction alternatives that reduce the probability of or the effects of soil liquefaction on the landfill cover are available for landfills where liquefaction of soil is predicted to disturb the landfill cover. In cases where soil liquefaction induced lateral movement or settlement is not predicted, there is no need to modify the design to protect against the effects of soil liquefaction. The potential for soil liquefaction and its effects, including lateral displacement and vertical settlement, are evaluated in this report for the Site 1 Landfill.

1.1 SCOPE AND ORGANIZATION OF DOCUMENT

This document presents the data and results of the liquefaction study for the Site 1 Landfill and areas immediately adjacent. The study involved review of existing data, collection of site-specific geotechnical field data, and assessment of the potential for liquefaction based on the site-specific data and conditions.

This report contains the following sections:

- Section 1.0 – Introduction. Describes the scope and organization of the document and the components and objective of the investigation.
- Section 2.0 – Subsurface Conditions. Discusses soil conditions at the site.
- Section 3.0 – Field Investigation Methods. Discusses the methods followed during the cone penetrometer tests (CPTs).
- Section 4.0 – Seismic Parameters. Discusses parameters and data gathered for the liquefaction evaluation.
- Section 5.0 – Liquefaction Potential and Soil Movement. Discusses in situ soil stresses and provides the analysis of liquefaction potential.
- Section 6.0 – Conclusions. Provides the conclusions from the evaluation of liquefaction potential at the site and mitigation measures.
- Section 7.0 – References. Lists the references used to prepare this report.
- Figures are presented after Section 7.0. Appendices that contain data and supporting information are presented following the figures.
- Appendix A contains the CPT logs.
- Appendix B presents the summary of soil liquefaction evaluation calculations.

1.2 OBJECTIVE AND COMPONENTS OF INVESTIGATION

The objective of this investigation was to complete a site-specific liquefaction study for the Site 1 Landfill. The field investigation obtained geological and engineering information that was used to evaluate the potential for liquefaction in soil under and adjacent to the Site 1 Landfill. In areas where some potential for liquefaction was indicated, the amount of consequent lateral soil movement and settlement was estimated.

1.2.1 Data Collection

Five CPTs were completed around the perimeter of the Site 1 Landfill (Figure 2) at locations selected to provide representative information for soil under and adjacent to the Site 1 Landfill. The CPTs were located and completed as described in the January 20, 2005, liquefaction study work plan (Tetra Tech 2005).

The CPTs provided information on the character and engineering properties of the soil. Soils were described by type, such as clay, silt, sand, or gravel; mixtures of several soil types were also identified. Selected engineering properties were also identified. For example, it was noted whether a soil was cohesive or cohesionless. Soil such as clay, in which the adsorbed water and soil particles form a relatively bonded mass, are known as cohesive soils. Soils that do not exhibit cohesion are termed cohesionless. Examples of cohesionless soil are sand and gravel without a significant clay fraction. Not all soil types are susceptible to liquefaction. Dense cohesionless soil and cohesive soil are generally not susceptible to liquefaction. Loose cohesionless soil that is saturated (below the groundwater level) can be susceptible to liquefaction.

Thicknesses of the soil layers and soil density were also recorded. The lateral extent of various types of soil deposits was assessed by comparing different CPT locations. Appendix A contains the CPT logs. Depth to groundwater was derived by reviewing hydrogeologic studies previously conducted at the Site 1 Landfill.

1.2.2 Earthquake Magnitude and Peak Ground Acceleration

Regardless of soil characteristics, liquefaction will not occur unless an earthquake shakes the ground with sufficient intensity. Specifically, the seismic waves must induce an intensity of ground acceleration sufficient to cause liquefaction for susceptible soils. This anticipated ground acceleration, and the earthquake that could cause it, were used in the evaluation of liquefaction.

The loading was predicted using a deterministic approach. As required by Title 27 CCR, a maximum probable earthquake (MPE) was used for seismic evaluation of municipal landfills. The MPE is either the earthquake that may occur in a 100-year recurrence interval or the largest historical earthquake. The MPE is expressed as a moment magnitude, which is based on the energy released by an earthquake. It is expressed on a logarithmic scale by a factor of 32, rather than of 10.

Once the MPE is identified, the peak ground acceleration (PGA) is estimated. Ground acceleration occurs in three dimensions, including horizontal and vertical components. The PGA in this report refers to the largest horizontal acceleration component of motion. Furthermore, the energy from an earthquake attenuates (decreases) with distance from the epicenter. The epicenter is the point on the surface of the earth above the focus of the earthquake, the spatial location of an earthquake within the earth's crust or mantle. Although ground acceleration generally attenuates with distance from the epicenter, the soil column may amplify the acceleration experienced by the underlying bedrock. Conversely, the soil column may attenuate the acceleration of the underlying bedrock. The relationship between the magnitude of an earthquake and PGA at distances from the epicenter was developed using values in Boore and others (1997), Campbell (1997), and Sadigh and others (1997) to calculate ground motion.

1.2.3 Evaluation of Potential for Liquefaction

The analytical methods used in this evaluation provide a basis to judge the likelihood of liquefaction, based on data obtained by researchers from historical liquefaction events. Researchers collected data from locations where liquefaction did and did not occur during earthquakes and identified the conditions that make liquefaction likely. Equations were then derived to predict the potential for liquefaction based on soil properties and anticipated PGA at a site.

Equations suitable for use with the CPT method of data collection were used in this evaluation. Technical discussion of the analyses used to estimate liquefaction potential may be found in Youd and others (2001) for data collected using the CPT method.

The general approach used to estimate liquefaction potential is known as the "cyclic stress approach" (Kramer 1996). The cyclic stress approach is conceptually simple: the earthquake-induced loading, expressed in terms of cyclic stresses, is compared with the resistance of the soil to liquefy, which is also expressed in terms of cyclic stresses. Liquefaction may occur at locations where the cyclic stress loading exceeds the cyclic stress resistance. Although conceptually simple, application of the cyclic stress approach requires careful attention to detail in characterizing loading conditions and resistance to liquefaction.

1.2.4 Lateral Soil Movement and Settlement

The shear strength of soil is lowered to the point that the soil may behave as a viscous fluid when liquefaction occurs. In this state, it is possible for liquefied soil to flow. However, soil will not always move when it liquefies. Youd and others (2002) used historical information from liquefaction-induced lateral movement of soil to develop equations to predict movement. Liquefaction-induced lateral soil movements can have significant ground-disturbing effects, so prediction of lateral movement of soil is important for any site where liquefaction of soil is considered likely.

Cohesionless soil grains can shift and settle during soil liquefaction. After an earthquake-induced soil liquefaction event, the soil grains in a loose deposit tend to come to rest in a closer and denser configuration than was present before the shaking began. The result is an increase in density and a corresponding decrease in the volume of the soil. A decrease in the volume of the soil layer can cause the ground surface to settle. Differences in the initial soil density or the thickness of loose soil layers can cause "differential settlement" in adjacent areas. In severe cases, differential settlement can cause large changes in ground surface elevations over short distances, damaging overlying structures, including landfill covers.

The amount of lateral soil movement caused by potential liquefaction at the Site 1 Landfill was estimated in this study. In addition, the amount of settlement at the ground surface that could result from densification of soil layers was estimated.

2.0 SUBSURFACE CONDITIONS

This section provides a brief overview of the subsurface conditions identified during the field investigation. This description of subsurface conditions is based on information from CPTs completed as part of this study. Plots that show soil stratigraphy are included in Appendix A. Most of the CPTs showed clay, silty clay, and clayey silt underlying the site. These cohesive soil types are not generally expected to liquefy during earthquakes. Therefore, these soils were not further evaluated in this study.

In contrast, data from CPT-01, CPT-02, and CPT-03 indicated silty and sandy soil interbeds in cohesive soil from about 6 to 12 feet below ground surface (bgs). Interbedded silt, sandy silt, sand, and silty sand were one to three feet in thickness. Within the clay were interspersed layers of silt, sandy silt, sand, and silty sand in CPT-03 between about 40 to 50 feet bgs. These interbeds were approximately 1 foot thick. The saturated silt, sandy silt, sand, and silty sand soil material types found in CPTs 01, 02, and 03, were considered potentially susceptible to liquefaction given sufficient ground acceleration. As a result, these materials were carried through the evaluation as described in the following sections of this report.

The cohesionless soil layers carried through the evaluation appeared discontinuous, which would preclude uniform development of liquefaction in the event that some or all of these layers undergo soil liquefaction during earthquake shaking.

The groundwater level at the site was estimated based on previous hydrogeologic studies at the site (Tetra Tech 1998). The groundwater level ranged from 1 to 5 feet bgs and was found to vary depending on the time of year. A groundwater level of 5 feet bgs was applied in the liquefaction evaluation. That is, saturated soil was assumed below 5 feet bgs.

3.0 FIELD INVESTIGATION METHODS

The investigation methods employed field testing to characterize the engineering properties of the soil. The field testing consisted of CPTs, conducted at five locations around the perimeter of the Site 1 Landfill (Figure 2) and designated CPT-01 through CPT-05. The CPT depths ranged from 55 to 70 feet bgs.

Gregg In Situ, Inc., of Martinez, California, completed the CPTs using an integrated electronic cone system. The truck-mounted integrated electronic cone system is specifically designed for CPTs. CPTs were carried out in general accordance with American Society for Testing and Materials (ASTM) Method D5778-95 (ASTM 1995). Appendix A provides the CPT logs.

The CPTs were completed using a 20-ton-capacity cone hydraulically pushed through the soil. The tip area of the cone was 15 square centimeters (cm^2) and the area of the friction sleeve was 225 cm^2. A 5-millimeter (mm)-thick piezometer element, located immediately behind the cone tip, measured the pressure of the water in the pore space of the soil. The term "stress" is used in lieu of "pressure" in geotechnical engineering practice. Both terms are used to represent force on a defined area (such as pounds per square foot). When the cone is pushed into the soil, the stress is partly applied to the soil grains and partly to the pore water. The stress applied to the soil grains can be estimated by the difference between the total stress and the stress in the pore water. The portion of stress that acts only on the soil grains is referred to as effective stress.

As the cone is pushed through the soil, instruments on the CPT rig record the following parameters:

- **Tip Resistance:** The force acting on the area of the tip as the cone is pushed into the soil

- **Sleeve Friction:** The shear force acting on the area of the sleeve as the cone is pushed

- **Dynamic Pore Pressure:** The pore water pressure at the tip as the cone is pushed

- **Penetration Depth:** The depth from the ground surface to the tip of the cone

- **Cone Angle:** The angle of the cone relative to vertical

- **Temperature:** Ground and groundwater temperature

These parameters were simultaneously printed and recorded electronically. The CPT data are presented in graphical form on the CPT logs, along with a computer-generated tabulation of interpreted soil type. Penetration depths are referenced to ground surface level at each CPT location.

The term "soil behavior type" (SBT) is used to interpret CPT data since direct observation of the soil is not possible. Measurements taken while the cone is advanced are used to infer SBT. The interpretation is based on relationships between cone tip resistance and sleeve friction, referred to as the "friction ratio" (Robertson and Campanella 1988). The friction ratio is a calculated parameter and is sleeve friction divided by tip resistance. The friction ratio is corrected for overburden pressure, since soil behaves differently under different confining stress.

Generally, cohesive soils have high friction ratios and low tip resistance. High pore water pressure is also generally measured in cohesive soil since its permeability is low. Cohesionless soils (sands) have lower friction ratios and higher tip resistance.

4.0 SEISMIC PARAMETERS

Important parameters that combine to create the potential for liquefaction in soil are earthquake magnitude, distance from the epicenter, PGA, soil characteristics, and the ability of the soil above bedrock to transmit horizontal acceleration. These parameters are defined below and in Section 1.2.3 of this report.

- **Magnitude:** The moment magnitude is based on the energy released by an earthquake. It is expressed on a logarithmic scale as a factor of 32, rather than of 10.

- **Epicenter:** The point on the surface of the earth above the focus of the earthquake, where the focus is the spatial location of an earthquake within the earth's crust or mantle, is the epicenter.

- **PGA:** The largest horizontal acceleration component of motion. The energy from an earthquake attenuates with distance. Correspondingly, the PGA will usually decrease with distance from the epicenter. In addition, the soil column may either amplify or attenuate the acceleration experienced by the underlying bedrock.

This section further discusses these parameters as related to the liquefaction potential study.

4.1 Seismicity and Faulting

Faults in the San Francisco Bay Region are of different lengths, slip rates, and types of movement. The types of movement in the San Francisco Bay Region are strike-slip and blind thrust, as described below:

- **Strike-Slip Fault:** In a strip-slip fault, one side of the fault moves horizontally relative to the other side.

- **Blind Thrust Fault:** A blind trust fault is a shallow-angle reverse fault without a surface trace. The fault plane lies at a shallow angle from the horizontal. The top side of the fault plane moves upward relative to the lower part.

The most common type of movement in the San Francisco Bay Region is the strike-slip. The rate of slip for the strike-slip-type faults ranges from about 2 to 24 mm per year. Over the long term, these faults release most of the seismic activity in the SFBR.

The Working Group on California Earthquake Probabilities (WG99, WG02) identified seven major faults of the San Andreas Fault system within 50 kilometers of the Site 1 Landfill (U.S. Geological Survey [USGS] 1999, 2003). These faults are understood to be capable of producing earthquakes of magnitude greater than or equal to 6.7 ($M \geq 6.7$), with the possible exception of the Calaveras Fault. There is uncertainty whether the Calaveras Fault can produce earthquakes of an $M \geq 6.7$ or whether it falls predominantly within the "moderate earthquakes and creep" category.

Fault creep is defined as slow, continued movement along a fault. Return intervals for moderate to large earthquakes on these seven faults average hundreds of years. Faults with lower slip rates located in the San Francisco Bay Region are also capable of producing moderate to large earthquakes. The return times for these earthquakes on faults with low slip rates are generally measured in thousands of years.

4.2 Earthquakes and Peak Ground Accelerations

Title 27 CCR requires that municipal landfill closure systems be designed to withstand the PGA from the MPE. The MPE is selected using a deterministic approach as either the earthquake that may occur in a 100-year recurrence interval or the largest historical earthquake. A M6.5 earthquake on the Concord fault was selected as the MPE (Mualchin 1996). A M6.5 represents an earthquake with approximately a 100-year recurrence interval.

The earthquake found to be the MPE from this deterministic approach has the following characteristics:

- Location: Concord Fault
- Magnitude: 6.5
- Distance from site: 5 kilometers

Based on these characteristics, the PGA estimated at the Site 1 Landfill was 0.45 gravity (g) using the attenuation relationship of Boore and others (1997).

5.0 LIQUEFACTION AND SOIL MOVEMENT

The following sections describe the analysis and results of the evaluation of liquefaction and soil movement. An M6.5 earthquake and a PGA of 0.45 were used in the analysis. A distance between the Site 1 Landfill and the earthquake epicenter of 5 kilometers was applied based on the distance from the Concord fault to the site.

5.1 LIQUEFACTION POTENTIAL

Analytical methods appropriate for data using CPTs were used in the evaluation. The method applied is presented in Youd and others (2001). Appendix B contains summaries of the calculations employed to evaluate liquefaction potential using data collected from CPTs.

Liquefaction is judged likely in this evaluation when the factor of safety is less than 1.2. The factor of safety is the ratio of (1) the strength of a soil to withstand liquefaction to (2) the forces acting to cause liquefaction. Theoretically, a factor of safety greater than or equal to 1.0 describes a condition under which soil liquefaction is not anticipated. Because the probability of soil liquefaction decreases as the factor of safety increases, it is common practice to add a 20 percent margin of safety to the evaluation. Thus, a factor of safety of 1.2 or greater is normally considered adequate (DMG 1997).

The evaluation of the factors of safety estimated for potentially liquefiable soil at the Site 1 Landfill ranged from less than 1.0 to 1.3. A PGA of 0.45g was applied. Factors of safety were calculated for discrete depth intervals of approximately 1 foot thick.

The estimated factors of safety for several layers ranging from about 1 to 3 feet thick were less than 1.0. These occur in three CPTs and at various depths as described below. Nowhere were uniform conditions found where soil liquefaction is considered likely in a laterally extensive or relatively thick soil deposit. The calculated liquefaction potential at various depths is discussed below in detail. Saturated soil to approximately 50 feet bgs, were evaluated for liquefaction potential. Below 50 feet bgs the analytical methods yield uncertain results (Youd and others 2001). Little empirical information is available for depths below 50 feet bgs because liquefaction below this depth seldom occurs.

5.1.1 Shallow Fill Soils Less Than 7 Feet Deep

In CPT-01 and CPT-02 the evaluation predicted factors of safety of less than 1.0 in cohesionless soils at depths of 6.5 to7.5-1/2 feet. These layers are composed of fill and do not represent the marshland foundation soils on which the landfill waste was placed. Because these shallow soils do not lie below the waste, they would not have an impact on the proposed cap.

5.1.2 Shallow Native Soil at 10.5 and 11.5 Feet bgs

A factor of safety of less than 1.0 was calculated for a cohesionless soil layer within soft bay mud in CPT-03. This soil layer was not found in any other CPT. The overall thickness is estimated to be less than 2 feet. Due the thinness and lateral discontinuity of this layer, liquefaction induced impact to the landfill cover is not indicated.

5.1.3 Native Soils Between 12 and 40 Feet

No cohesionless soils were indicated within this depth interval.

5.1.4 Native Soils Between Depths of 40 and 50 Feet

Within the depth interval of 40 to 50 feet, soil types that are potentially susceptible to liquefaction were detected only in CPT-03. Interbedding of sandy silt and silty sand was noted within this depth interval in CPT-03. Of the 10 1-foot layers in that interval, 7 had predicted factors of safety of less than 1.0, indicating potential for liquefaction. One layer had a predicted factor of safety of 1.3, and two layers were of cohesive soil types. No other CPTs detected cohesionless soils within the depth interval of 40 to 50 feet.

5.2 LATERAL MOVEMENT

Lateral soil movement was evaluated using the analytical method for sloping ground conditions (Youd and others 2002). The method was developed based on empirical data from sites where lateral spread displacement was not impeded by shear or compression forces along the margins or at the toe of the lateral spread.

The westerly facing portion of the cover surface will be sloped at about 3 percent to allow for surface water to drain. Although the slope will be steeper along the eastern portion, the eastern slope is less extensive and is buttressed by the surrounding roadway. The slope of the cover at 3 percent would have little, if any, effect on soil at 40 to 50 feet bgs. Therefore, no slope was applied in the assessment of lateral movement at these depths. The ground surface above potentially liquefiable soil located from 5 to 10 feet bgs would be relatively flat.

The following parameters were used to estimate lateral movement:

- Moment magnitude of earthquake (M): M6.5

- Horizontal distance to the site from the earthquake (R): R = 5 kilometers

- Cumulative thickness of soil layer with corrected SPT blow counts less than 15 (T15): Varied; estimated for individual exploration locations

- Fines content of soil (fraction of soil passing a U.S. Standard No. 200 sieve) for granular soil materials included in T15 (F15): Varied based on soil type

- The average mean grain size for granular materials within T15 (D50 15): Varied based on soil type

- The ground slope (S): S = 0%

The evaluation suggests that no lateral movement of soil is likely at the Site 1 Landfill as a result of soil liquefaction during earthquakes.

Standard penetration test (SPT) blow counts were estimated from CPT data. SPT blow counts were not, however, used to estimate the potential for liquefaction. SPT values estimated from CPT data are not considered reliable for assessing the potential for liquefaction (Youd and others 2001). The SPT values were estimated only to provide a basis to evaluate lateral movement of soil and ground settlement.

5.3 SOIL SETTLEMENT

The analytical method described in Tokimatsu and Seed (1987) was used to estimate ground settlement. This method uses SPT blow counts to represent the density of soil. In this study, SPT blow counts were estimated using CPT data. Note that SPT blow counts were only applied to estimate potential settlement and were not applied to evaluate liquefaction potential.

The evaluation of soil settlement suggests that no ground surface settlement is likely at the Site 1 Landfill as a result of liquefaction during earthquakes. However, settlement caused by decay and consolidation of waste, unrelated to soil liquefaction, is still expected. Consolidation settlement and settlement caused by waste decay were considered in the cover design.

6.0 CONCLUSIONS

The geotechnical field investigation resulted in sufficient data to allow evaluation of the potential for liquefaction, lateral soil movement, and associated settlement at the Site 1 Landfill. Estimated factors of safety indicated a potential for liquefaction of several discrete native soil layers below the waste. Of these, most were located at depths where liquefaction is less likely. Because of the degree of inter-bedding of soil layers and lack of layer continuity, uniform liquefaction of multiple layers and extensive soil areas is not considered likely below the Site 1 Landfill.

An evaluation was performed to separately consider the possibility of lateral movement and settlement in the event the soil liquefaction did occur. The evaluation of lateral movement during earthquake induced liquefaction suggests that lateral movement will not occur below the waste. Likewise, settlement of the cover related to soil liquefaction is not predicted. Because lateral movement and settlement are not predicted as a result of earthquake inducted liquefaction, the cover will not require any special design features to accommodate earthquake induced movements.

Tetra Tech EM Inc. (Tetra Tech) completed the evaluations described in this report consistent with generally accepted professional consulting principles and practices. Professional judgment was applied. No other warranty, express or implied, is made.

Opinions and recommendations contained in this report apply to conditions existing, or presumed to exist based upon field data collected, when services were rendered and are intended only for the client, purposes, locations, and project parameters indicated. Tetra Tech is not responsible for the effects of any changes in standards, practices, or regulations subsequent to performance of services. Tetra Tech does not warrant the accuracy of information supplied by others, or the use of segregated portions of this report.

7.0 REFERENCES

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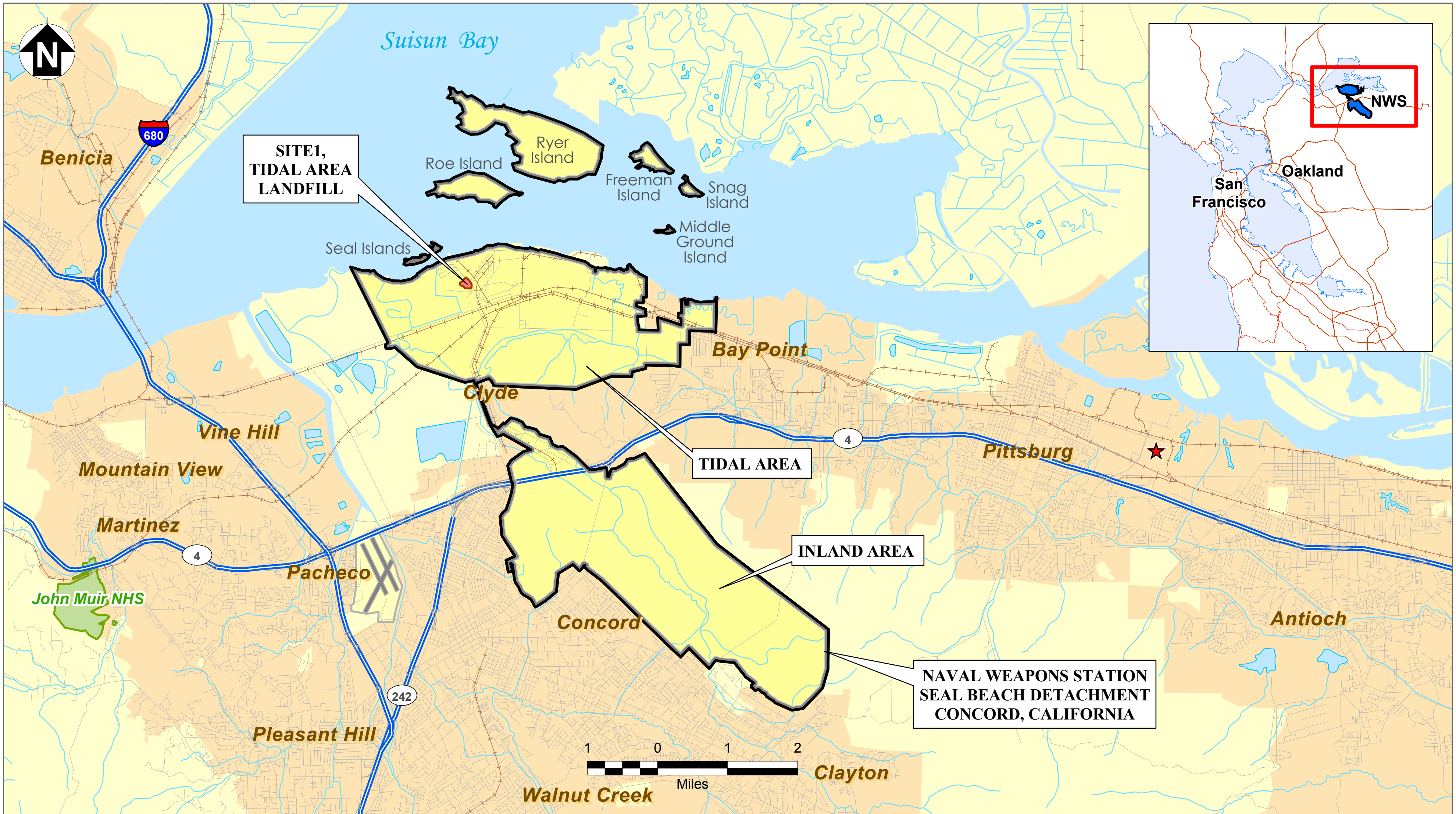
FIGURES



Naval Weapons Station Seal Beach Detachment Concord, California

Tidal Area Landfill

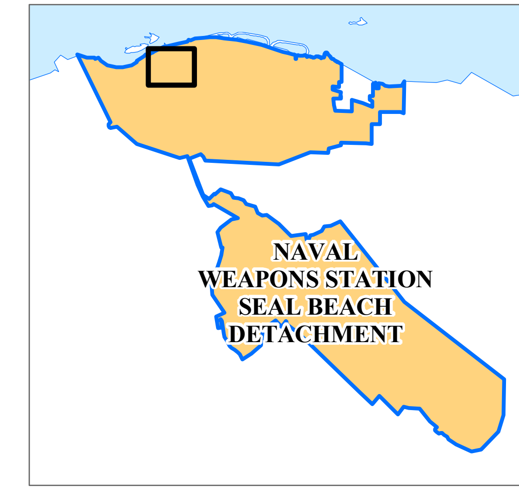
★ Radiography Facility

Tetra Tech EM Inc.

Naval Weapons Station Seal Beach Detachment
Concord, California
Integrated Product Team West, Daly City

FIGURE 1
GENERAL VICINITY PLAN
LOCATION MAP

Liquefaction Study Work Plan



CPT Location

Site Boundary

Notes:
1. CPT Cone Penetrometer Location.
2. Source of aerial photograph:
http://www.acme.com/mapper

N

300 0 300
Feet

Tetra Tech EM Inc.

Naval Weapons Station Seal Beach Detachment
Concord, California
Integrated Product Team West, Daly City

FIGURE 2
SITE PLAN SHOWING CPT LOCATIONS
SITE 1, TIDAL AREA LANDFILL

Liquefaction Study Work Plan

2005-04-07 V:\Concord\Projects\Tidal_Liquefaction_Study\CPT_Locs.mxd TtEMI-SF Aleksandr Zhuk

APPENDIX A
CPT TEST DATA

Cone Penetration Testing Procedure (CPT)

Gregg In Situ, Inc. carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and dynamic pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT*. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain dynamic pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

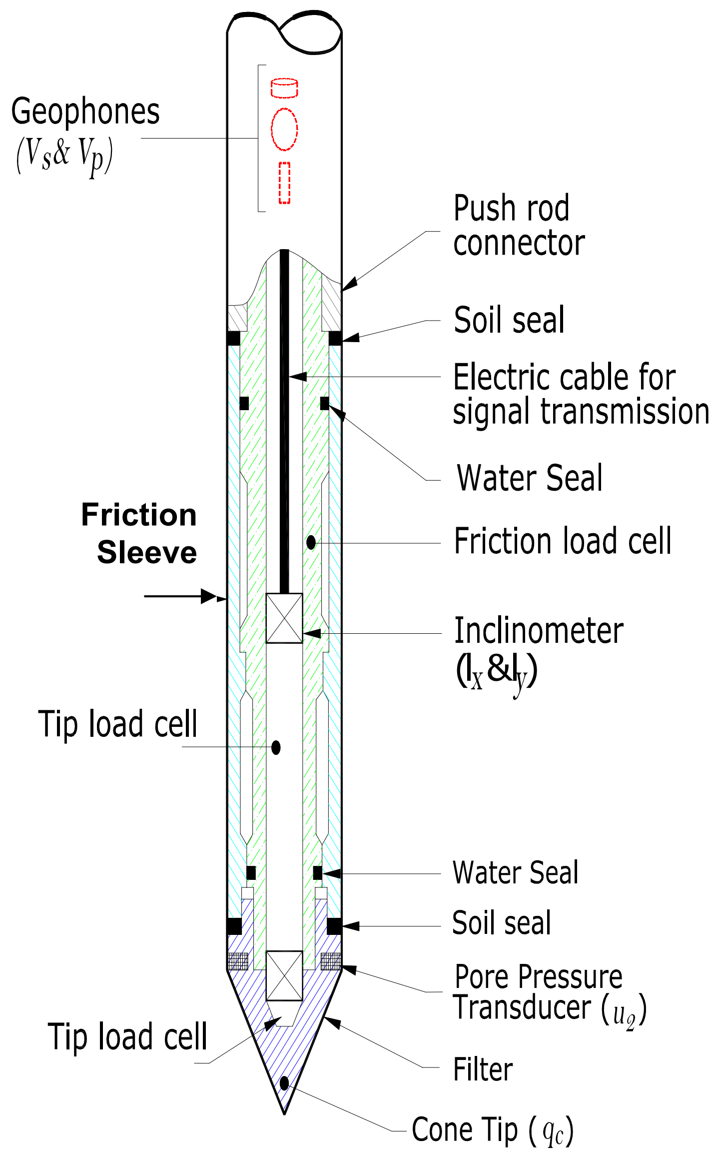


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg In Situ support rig. The grouting procedure consists of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

Cone Penetration Test Data & Interpretation

Soil behavior type and stratigraphic interpretation is based on relationships between cone bearing (q_c), sleeve friction (f_s), and pore water pressure (u_2). The friction ratio (R_f) is a calculated parameter defined by $100 f_s / q_c$ and is used to infer soil behavior type. Generally:

Cohesive soils (clays)

- High friction ratio (R_f) due to small cone bearing (q_c)
- Generate large excess pore water pressures (u_2)

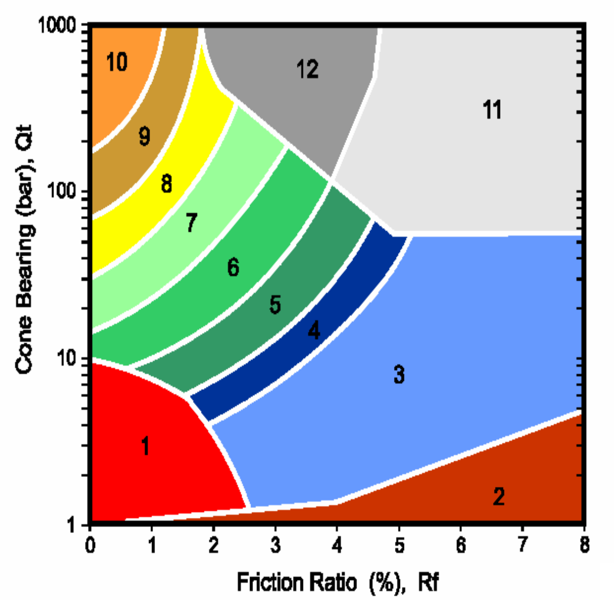
Cohesionless soils (sands)

- Low friction ratio (R_f) due to large cone bearing (q_c)
- Generate very little excess pore water pressures (u_2)

A complete set of baseline readings are taken prior to and at the completion of each sounding to determine temperature shifts and any zero load offsets. Corrections for temperature shifts and zero load offsets can be extremely important, especially when the recorded loads are relatively small. In sandy soils, however, these corrections are generally negligible.

The cone penetration test data collected from your site is presented in graphical form in Appendix CPT. The data includes CPT logs of measured soil parameters, computer calculations of interpreted soil behavior types (SBT), and additional geotechnical parameters. A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Soil interpretation for this project was conducted using recent correlations developed by Robertson et al, 1990, *Figure SBT*. Note that it is not always possible to clearly identify a soil type based solely on q_c, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.



ZONE	Qt/N	SBT
1	2	Sensitive, fine grained
2	1	Organic materials
3	1	Clay
4	1.5	Silty clay to clay
5	2	Clayey silt to silty clay
6	2.5	Sandy silt to clayey silt
7	3	Silty sand to sandy silt
8	4	Sand to silty sand
9	5	Sand
10	6	Gravely sand to sand
11	1	Very stiff fine grained*
12	2	Sand to clayey sand*

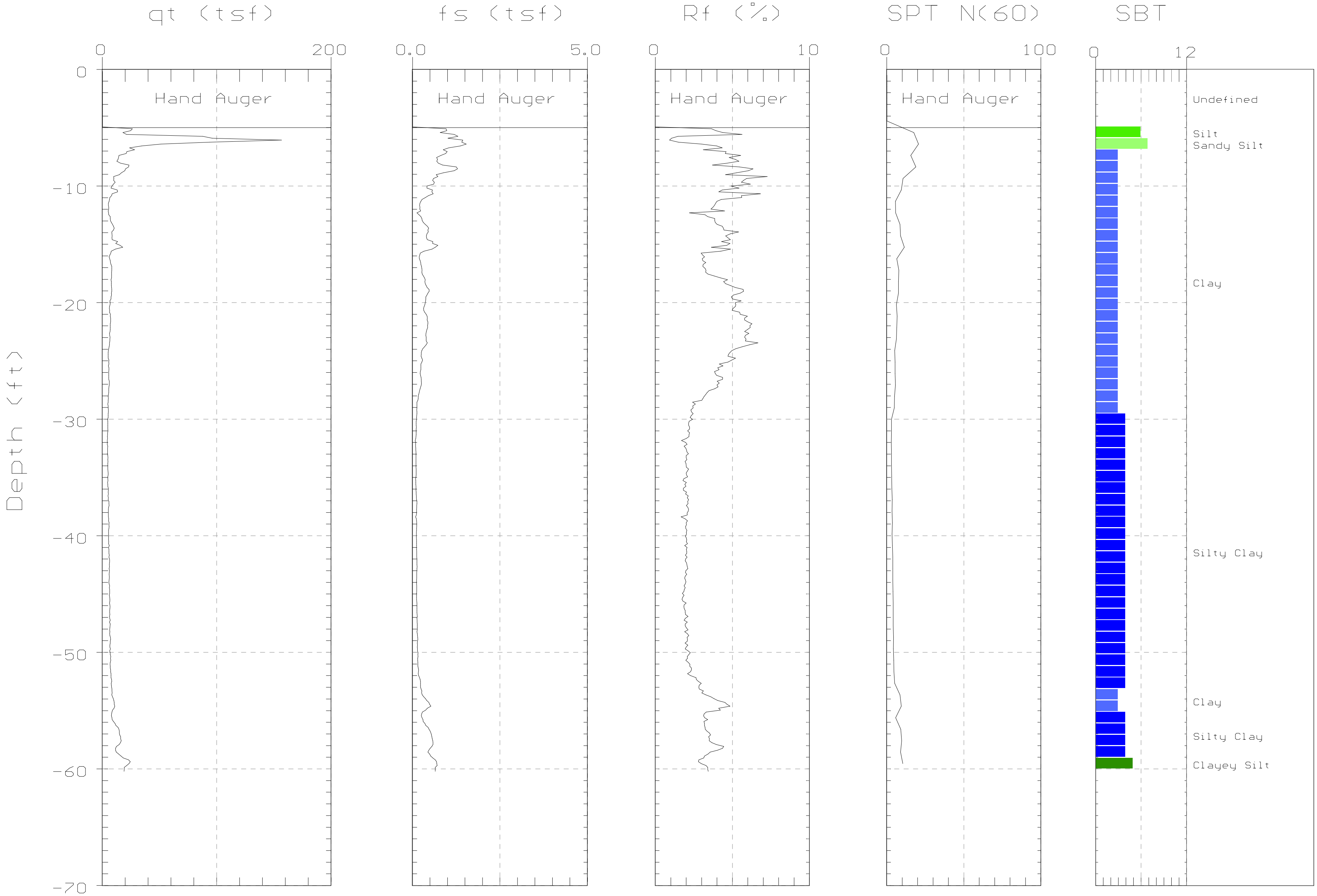
*over consolidated or cemented

Figure SBT

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-01

Engineer: P.CALLAHAN
Date: 01:26:05 09:25



Max. Depth: 60.20 (ft)
Depth Inc.: 0.164 (ft)

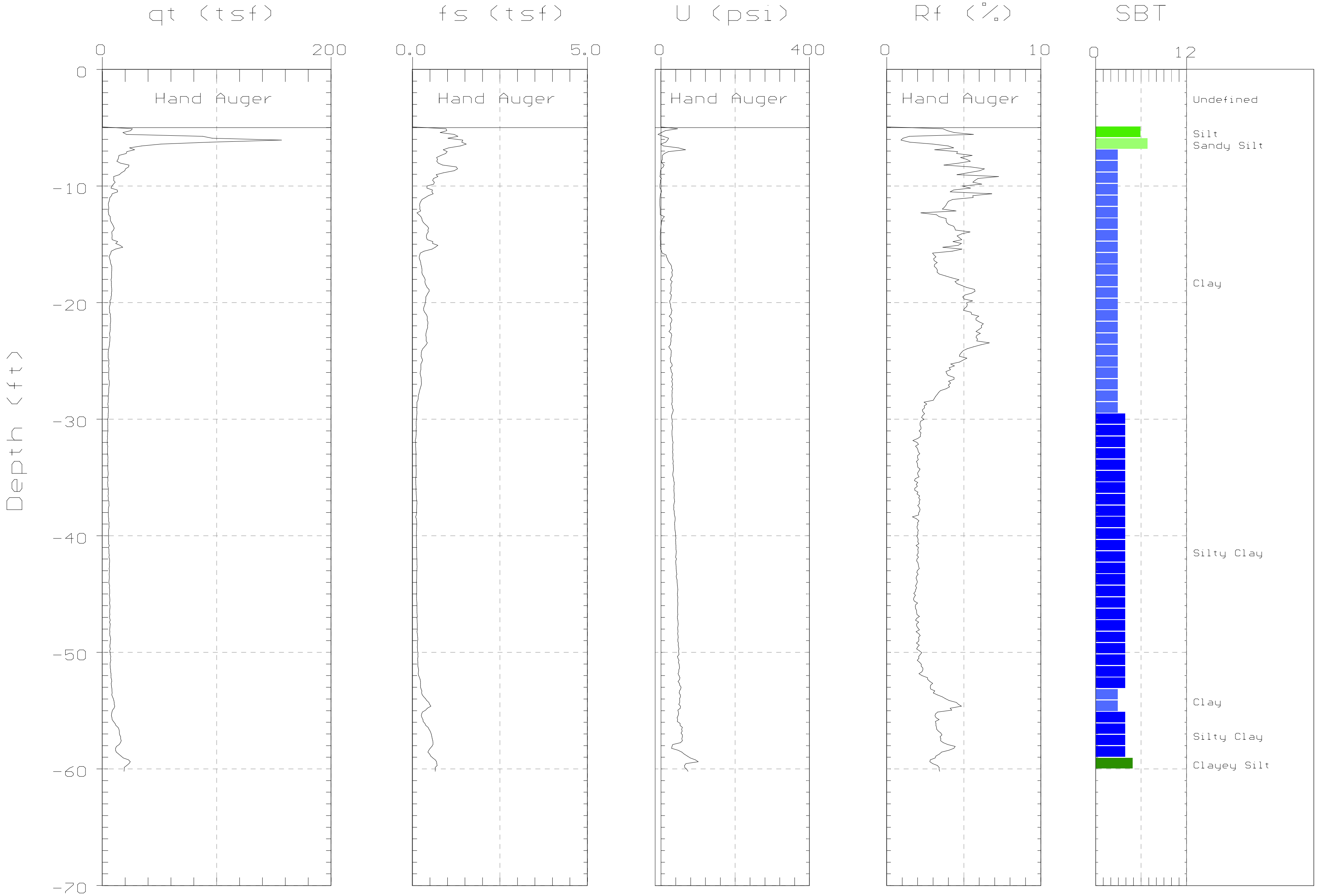
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-01

Engineer: P.CALLAHAN
Date: 01:26:05 09:25



Max. Depth: 60.20 (ft)
Depth Inc.: 0.164 (ft)

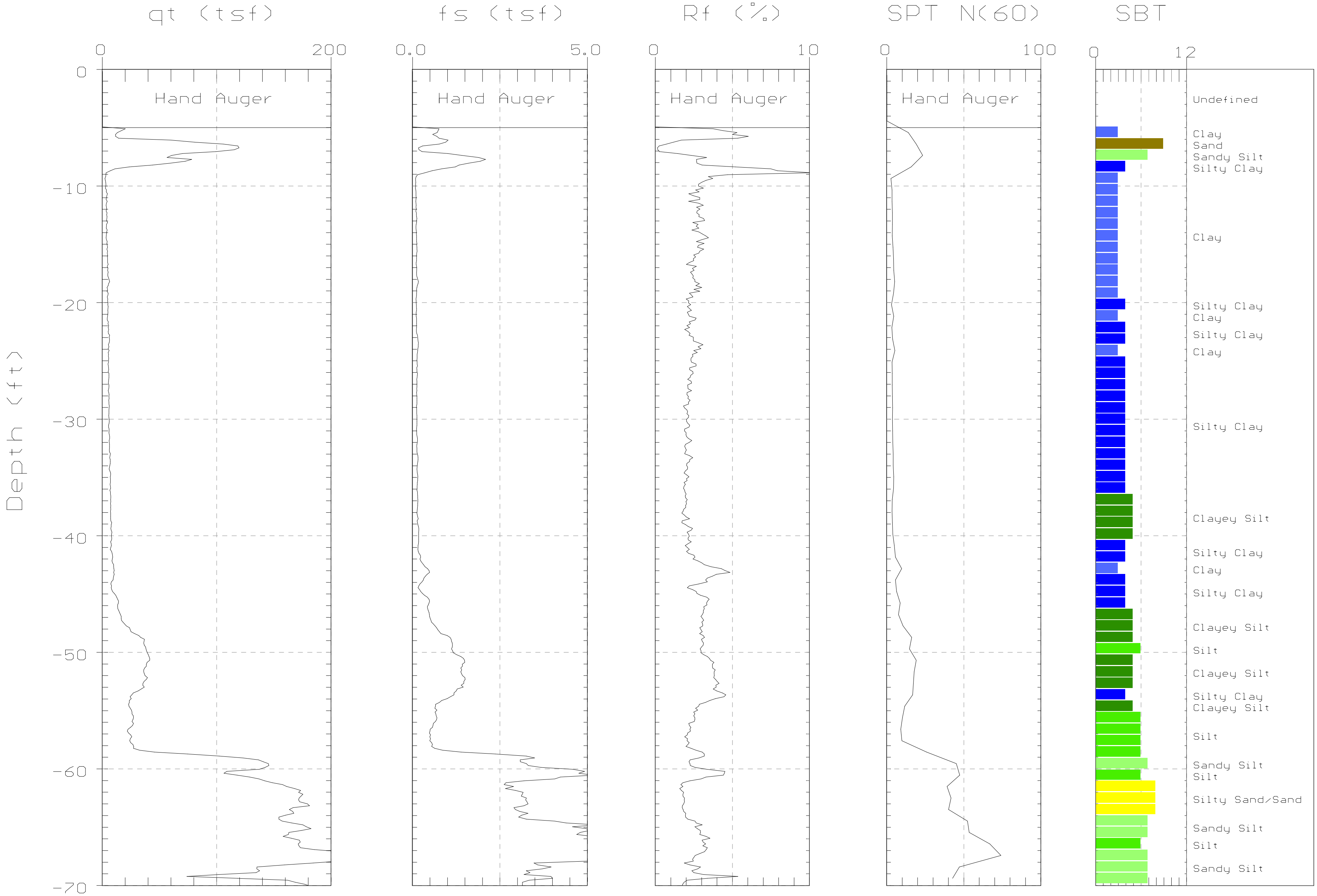
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

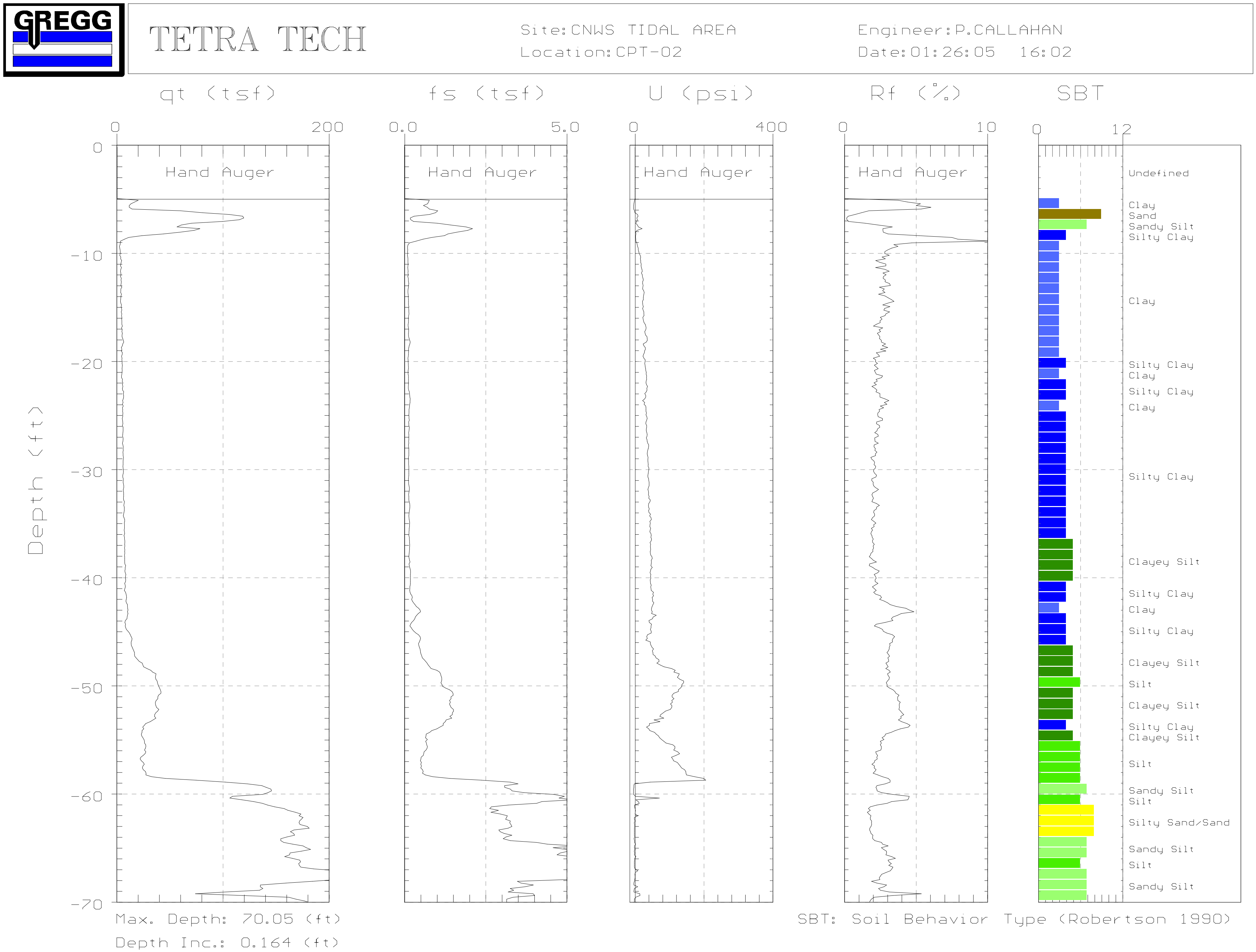
Site: CNWS TIDAL AREA
Location: CPT-02

Engineer: P.CALLAHAN
Date: 01:26:05 16:02



Max. Depth: 70.05 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

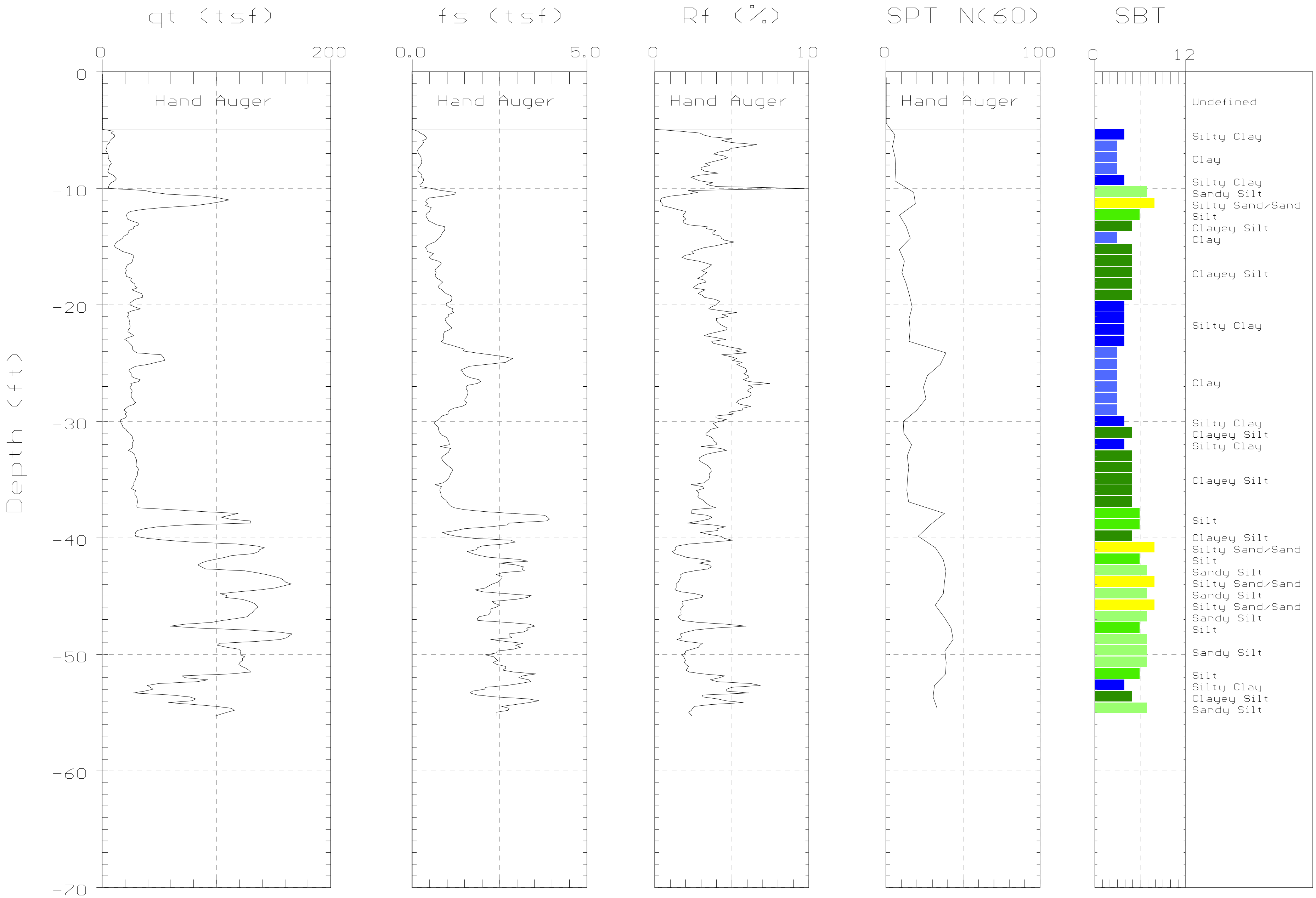

GREGG
TETRA TECH
Site: CNWS TIDAL AREA
Location: CPT-02
Engineer: P.CALLAHAN
Date: 01:26:05 16:02
qt (tsf)
fs (tsf)
U (psi)
Rf (%)
SBT
Hand Auger
Depth (ft)
Undefined
Clay
Sand
Sandy Silt
Silty Clay
Clayey Silt
Silt
Silty Sand/Sand
Max. Depth: 70.05 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-03

Engineer: P.CALLAHAN
Date: 01:26:05 12:00



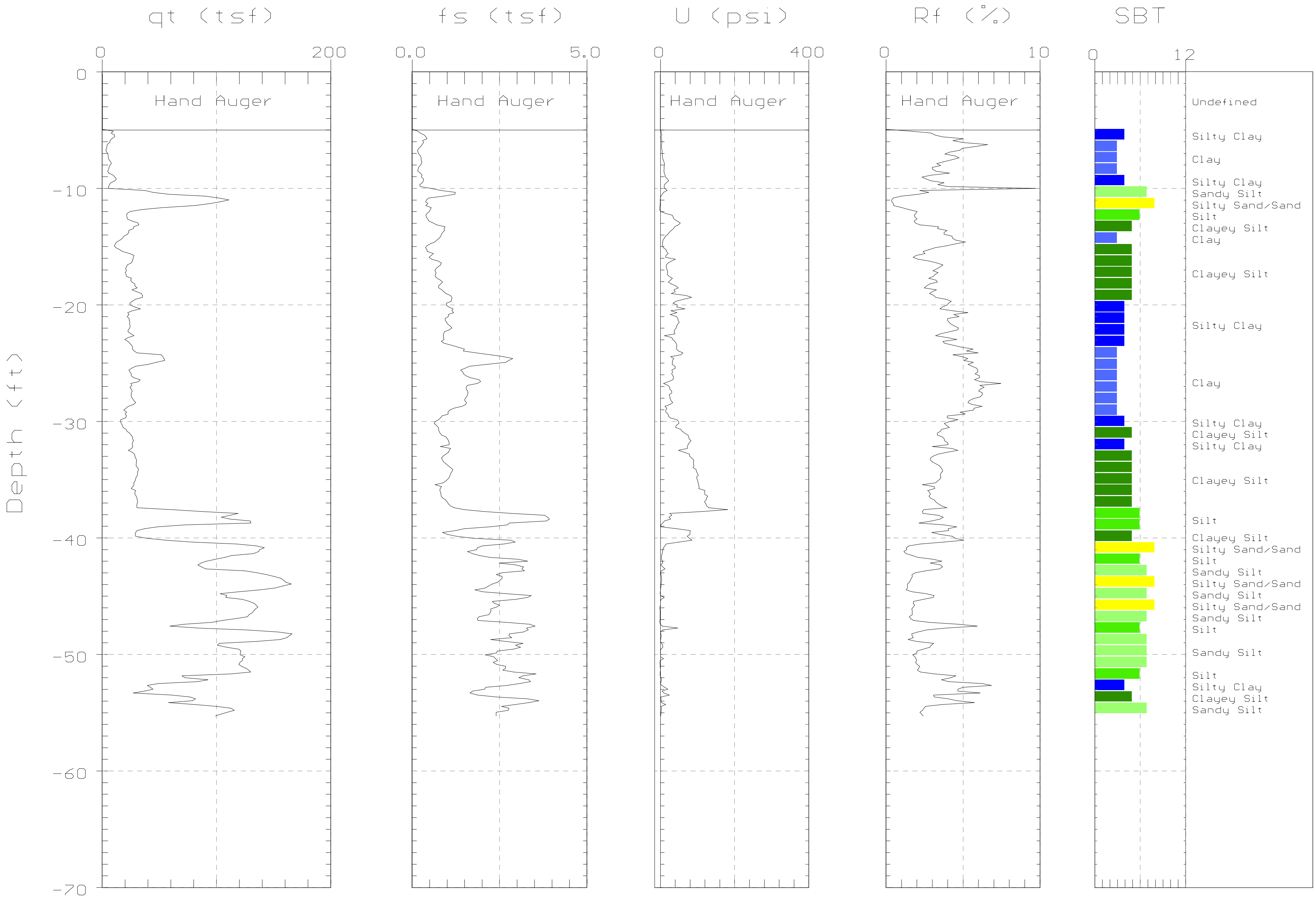
Max. Depth: 55.28 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-03

Engineer: P.CALLAHAN
Date: 01:26:05 12:00



Max. Depth: 55.28 (ft)
Depth Inc.: 0.164 (ft)

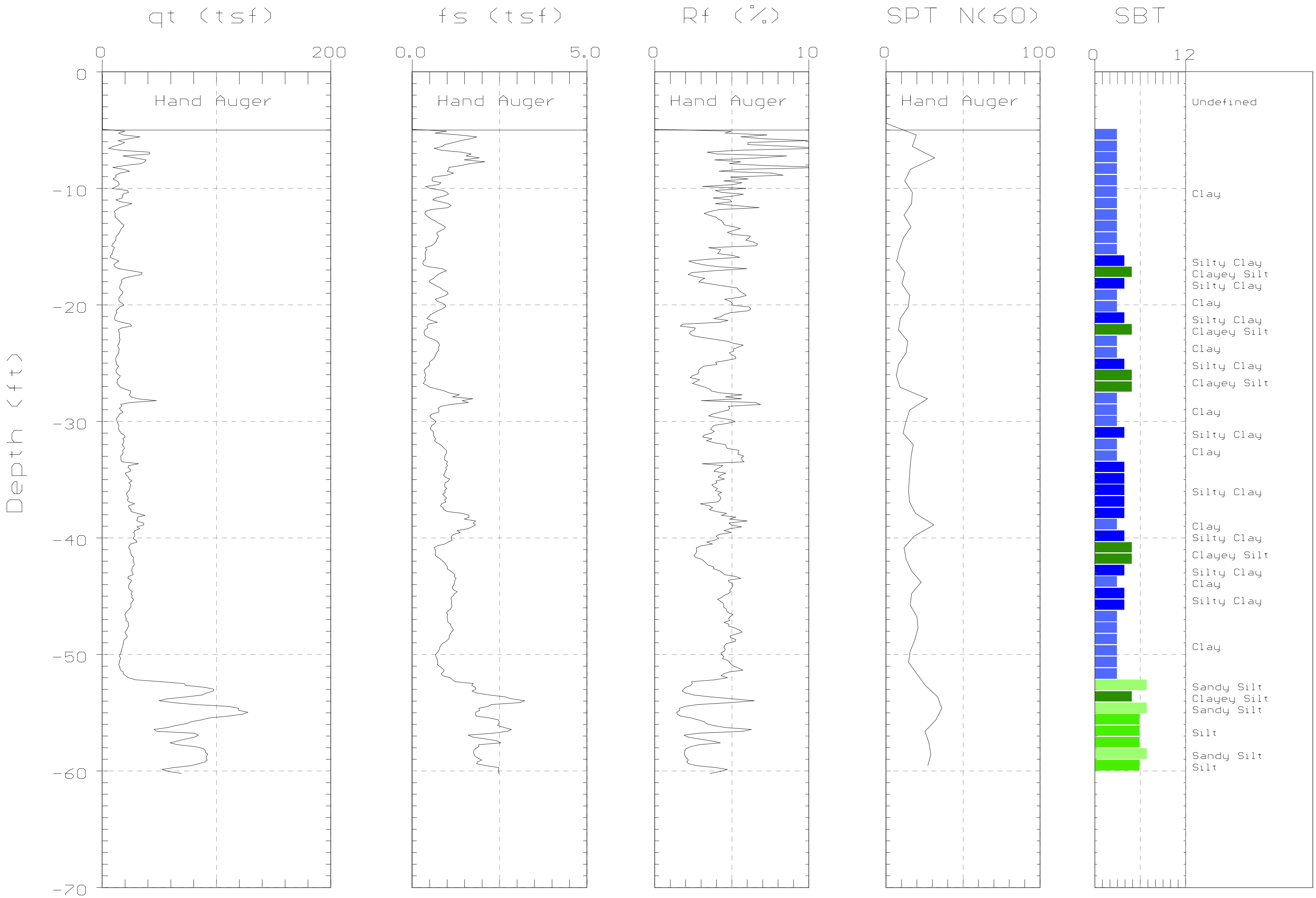
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-04

Engineer: P.CALLAHAN
Date: 01:26:05 13:08



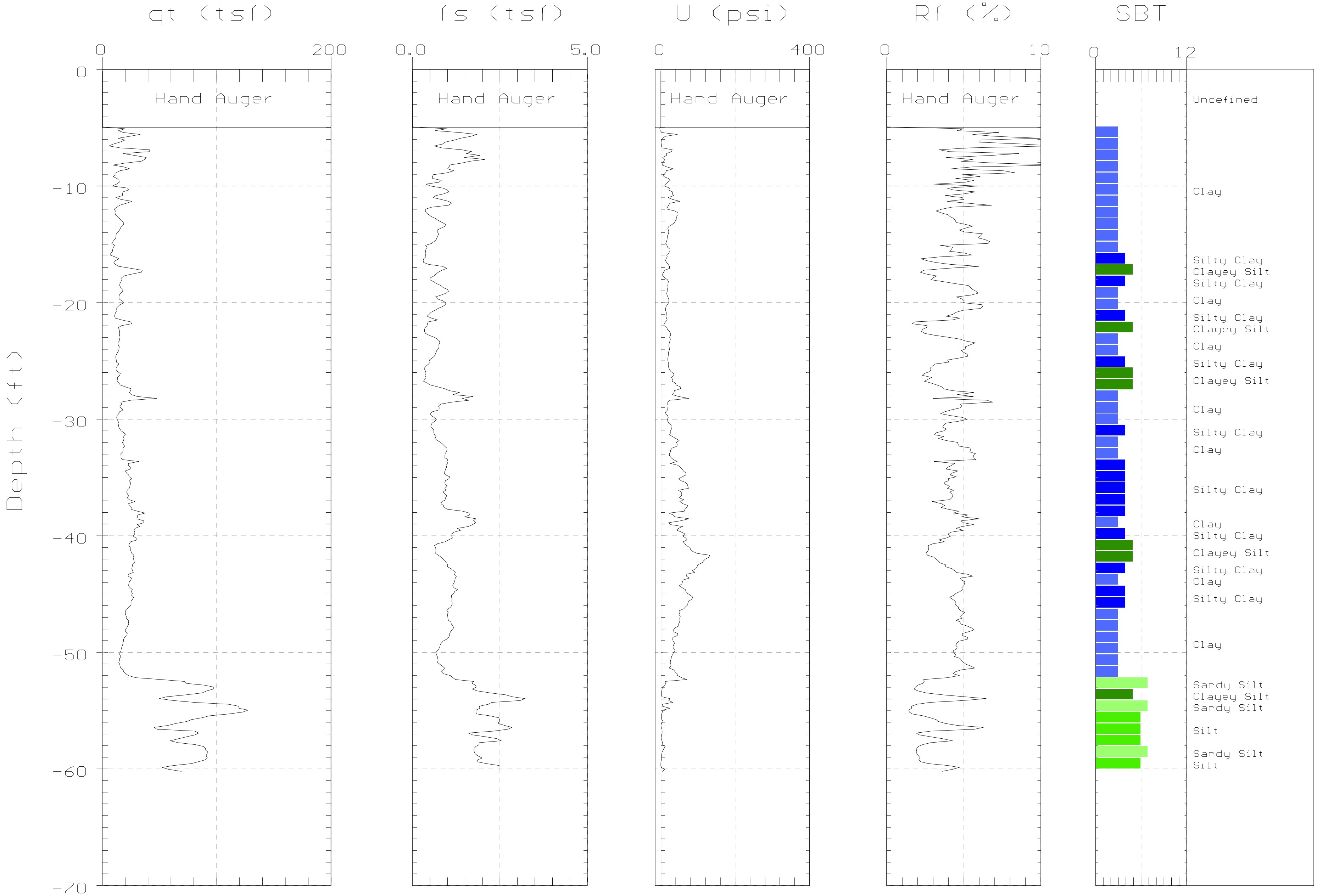
Max. Depth: 60.20 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-04

Engineer: P.CALLAHAN
Date: 01:26:05 13:08



Max. Depth: 60.20 (ft)
Depth Inc.: 0.164 (ft)

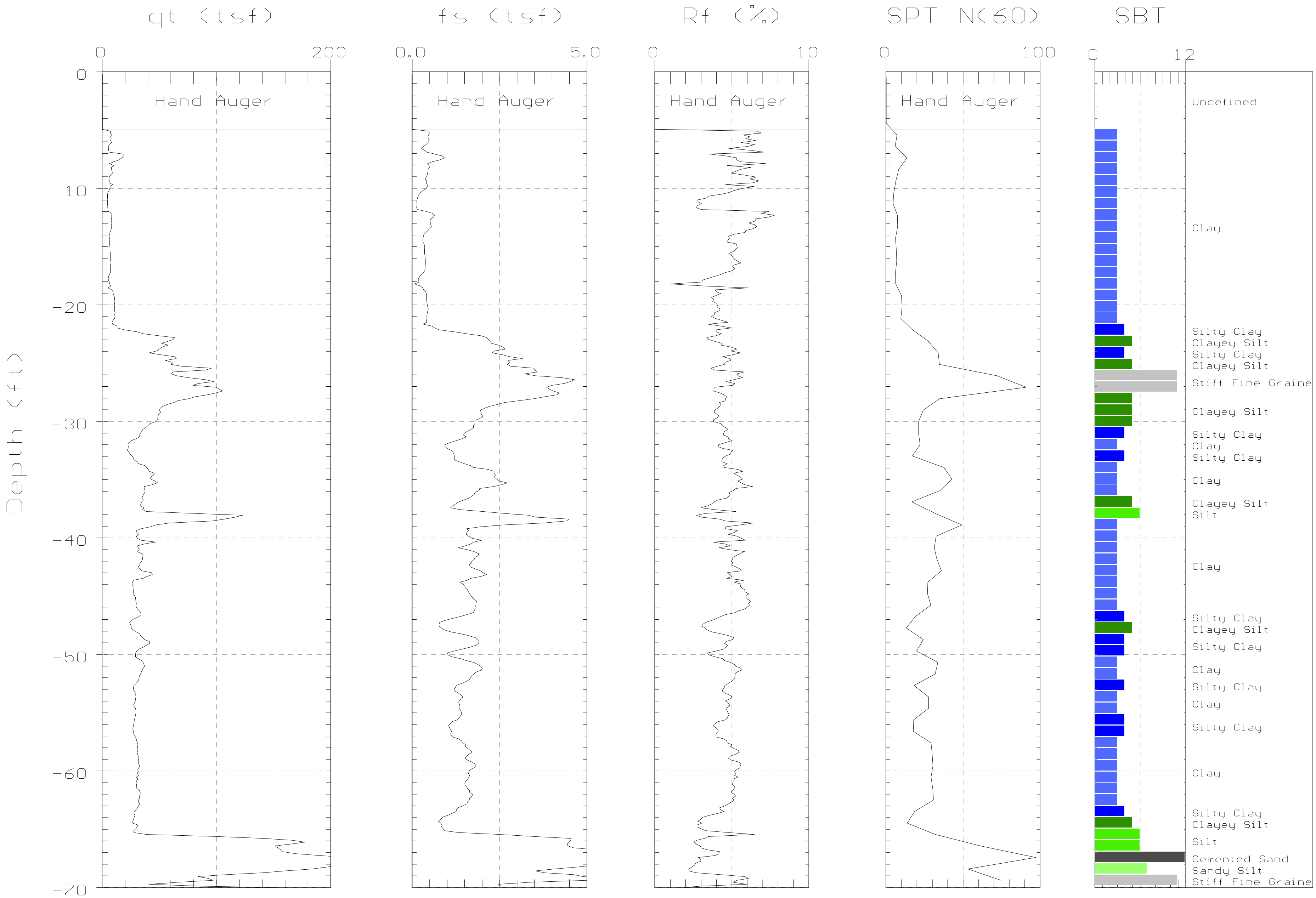
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-05

Engineer: P.CALLAHAN
Date: 01:26:05 14:49



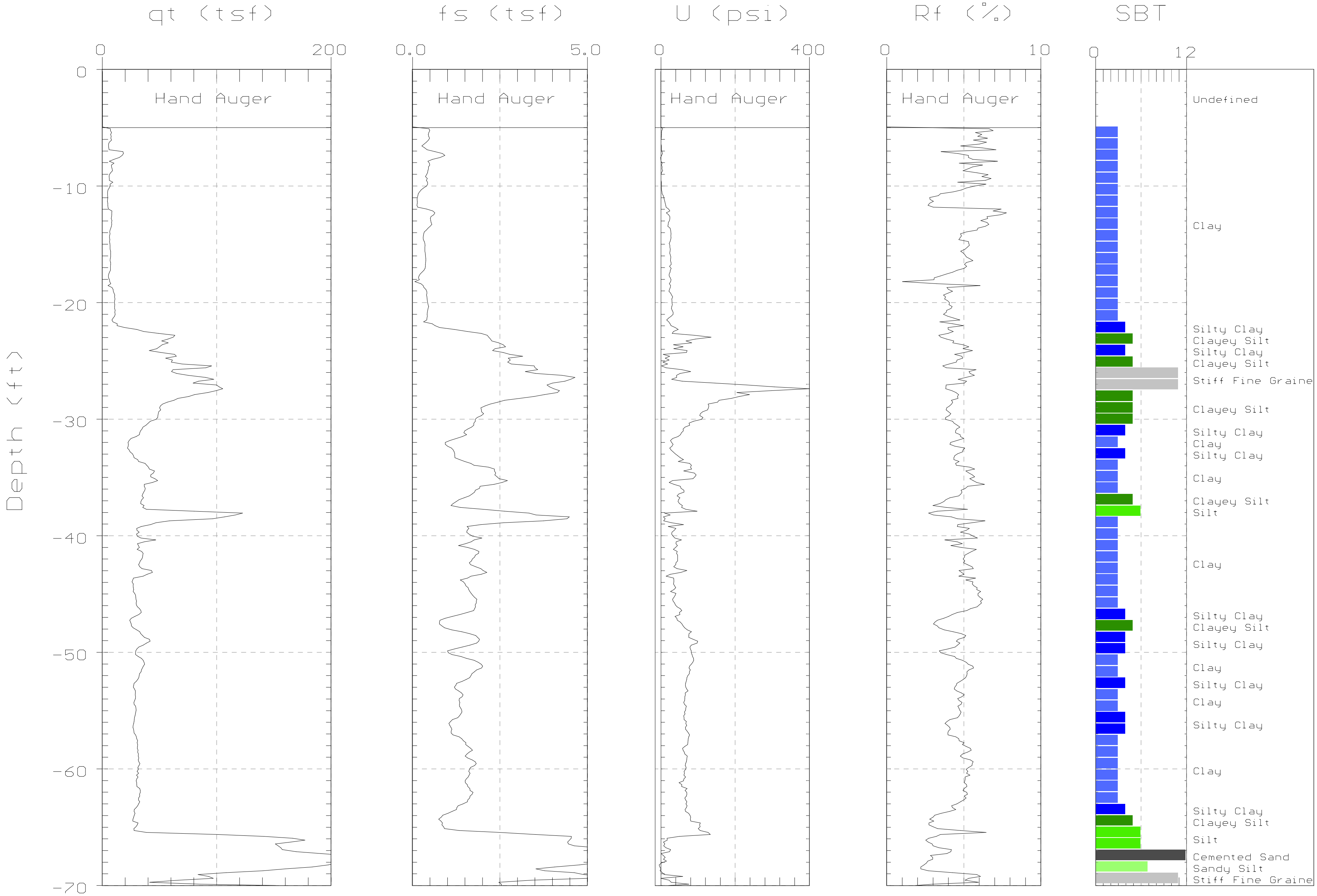
Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TETRA TECH

Site: CNWS TIDAL AREA
Location: CPT-05

Engineer: P.CALLAHAN
Date: 01:26:05 14:49



Max. Depth: 70.21 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B
SUMMARY OF SOIL LIQUEFACTION EVALUATION CALCULATIONS

Project Tidal Area Landfill **Date** Feb. 10, 2004
Client NAVFAC SWDIV

Soil Type	Description	Fines Content, %	D_{50} mm	Dry Density, pcf	Moisture Content,%
1	Sensitive Fine Grain	99	0.02	80	15
2	Organic	99	--	80	25
3	Clay	99	--	110	20
4	Silty Clay - Clay	99	--	115	20
5	Clayey Silt - Silty Clay	99	--	115	20
6	Sandy Silt - Clayey Silt	80	--	115	15
7	Silty Sand - Sandy Silt	50	0.2	115	10
8	Sand - Silty Sand	20	0.3	120	10
9	Sand	5	0.4	125	10
10	Gravelly Sand - Sand	5	--	125	5
11	V Stiff Fine Grain/Over Con	99	--	130	20
12	Sand - Clayey Sand/Over Con	50	--	120	15

Design Magnitude	6.5	6.0 to 8.5
R, km	5	Distance from seismic energy source
Ground Acceleration, g	0.45	

Exploration No CPT-01 **Date Completed** Feb. 10, 2004

Comments:

Depth to Groundwater, feet 5

Horizontal Displacement

Height of Nearest Slope Face, feet	0
Distance from Slope Face, feet	0
Ground Surface Grade, %	3%
Depth to Top of Layer of Concern	0
Depth to Bottom of Layer of Concern	1
Max. Displacement, ft	0

Depth feet	**SPT** $N_{1\,60}$	**SBT**	Q_{c1ncs}	**Response Acceleration, g**	**Vs** m/sec	**FS** Q_{c1ncs}	**FS Vs**	**Total Settlement**, in	**Horiz. Displace. Sloping Ground**, ft
0.5				0.45					
1.5				0.45					
2.5				0.45					
3.4				0.45					
4.4				0.45					
5.4	31	6	46	0.45					
6.4	33	7	64	0.45		<1			0
7.4	23	3	16	0.45					
8.4	27	3	20	0.45					
9.4	14	3	11	0.45					
10.3	12	3	10	0.45					
11.3	7	3	6	0.45					
12.3	7	3	6	0.45					
13.3	10	3	9	0.45					
14.3	10	3	9	0.45					
15.3	12	3	12	0.45					
16.2	7	3	7	0.45					
17.2	8	3	8	0.45					
18.2	7	3	8	0.45					
19.2	7	3	8	0.45					
20.2	6	3	6	0.45					
21.2	6	3	7	0.45					
22.1	6	3	7	0.45					
23.1	5	3	6	0.45					
24.1	4	3	5	0.45					
25.1	4	3	5	0.45					
26.1	4	3	5	0.45					
27.1	4	3	6	0.45					
28.1	4	3	5	0.45					
29.0	4	3	5	0.45					
30.0	2	4	5	0.45					
31.0	2	4	4	0.45					
32.0	2	4	4	0.45					
33.0	2	4	4	0.45					
34.0	2	4	4	0.45					
34.9	2	4	5	0.45					
35.9	2	4	5	0.45					
36.9	2	4	5	0.45					
37.9	2	4	5	0.45					
38.9	2	4	5	0.45					
39.9	2	4	5	0.45					
40.8	2	4	5	0.45					
41.8	3	4	6	0.45					
42.8	2	4	6	0.45					
43.8	2	4	6	0.45					
44.8	3	4	6	0.45					
45.8	3	4	6	0.45					
46.8	3	4	6	0.45					
47.7	3	4	6	0.45					
48.7	3	4	6	0.45					
49.7	3	4	6	0.45					
50.7	3	4	7	0.45					
51.7	3	4	7	0.45					
52.7	3	4	8	0.45					
53.6	5	3	8	0.45					
54.6	5	3	9	0.45					
55.6	3	4	9	0.45					
56.6	5	4	14	0.45					
57.6	5	4	15	0.45					
58.6	5	4	13	0.45					
59.5	6	5	21	0.45					

Exploration No CPT-02			**Date Completed** Feb. 10, 2004
Comments:			
		Horizontal Displacement	
		Height of Nearest Slope Face, feet	0
Depth to Groundwater, feet 5		Distance from Slope Face, feet	0
		Ground Surface Grade, %	0%
		Depth to Top of Layer of Concern	0
		Depth to Bottom of Layer of Concern	1
		Max. Displacement, ft	0

Depth feet	**SPT** $N_{1\,60}$	**SBT**	Q_{c1ncs}	**Response Acceleration, g**	**Vs** m/sec	**FS** Q_{c1ncs}	**FS Vs**	**Total Settlement**, in	**Horiz. Displace. Sloping Ground**, ft
0.5				0.45					
1.5				0.45					
2.5				0.45					
3.4				0.45					
4.4				0.45					
5.4	25	3	15	0.45					
6.4	31	9	100	0.45		<1			0
7.4	35	7	73	0.45		<1			0
8.4	22	4	25	0.45					
9.4	4	3	3	0.45					
10.3	4	3	3	0.45					
11.3	4	3	3	0.45					
12.3	4	3	4	0.45					
13.3	4	3	4	0.45					
14.3	4	3	3	0.45					
15.3	4	3	4	0.45					
16.2	5	3	4	0.45					
17.2	5	3	4	0.45					
18.2	5	3	5	0.45					
19.2	4	3	4	0.45					
20.2	3	4	5	0.45					
21.2	4	3	4	0.45					
22.1	3	4	5	0.45					
23.1	3	4	6	0.45					
24.1	4	3	5	0.45					
25.1	3	4	5	0.45					
26.1	3	4	5	0.45					
27.1	3	4	5	0.45					
28.1	3	4	5	0.45					
29.0	3	4	5	0.45					
30.0	3	4	5	0.45					
31.0	3	4	6	0.45					
32.0	3	4	6	0.45					
33.0	3	4	6	0.45					
34.0	3	4	6	0.45					
34.9	3	4	7	0.45					
35.9	3	4	7	0.45					
36.9	2	5	7	0.45					
37.9	2	5	7	0.45					
38.9	2	5	7	0.45					
39.9	3	5	8	0.45					
40.8	3	4	7	0.45					
41.8	4	4	8	0.45					
42.8	6	3	10	0.45					
43.8	4	4	8	0.45					
44.8	4	4	9	0.45					
45.8	5	4	13	0.45					
46.8	5	5	15	0.45					
47.7	6	5	21	0.45					
48.7	10	5	33	0.45					
49.7	9	6	37	0.45					
50.7	11	5	39	0.45					
51.7	10	5	36	0.45					
52.7	10	5	35	0.45					
53.6	9	4	26	0.45					
54.6	6	5	24	0.45					
55.6	6	6	26	0.45					
56.6	5	6	23	0.45					
57.6	5	6	24	0.45					
58.6	14	6	66	0.45					
59.5	24	7	141	0.45		<1			0

Exploration No CPT-03 **Date Completed** Feb. 10, 2004
Comments:

Depth to Groundwater, feet 5

Horizontal Displacement

Height of Nearest Slope Face, feet	0
Distance from Slope Face, feet	0
Ground Surface Grade, %	0%
Depth to Top of Layer of Concern	0
Depth to Bottom of Layer of Concern	1
Max. Displacement	0

Depth feet	**SPT** $N_{1\,60}$	**SBT**	Q_{c1ncs}	**Response Acceleration, g**	**Vs** m/sec	**FS** Q_{c1ncs}	**FS Vs**	**Settlement** inch	**Horiz. Displace. Sloping Ground**, ft
0.5				0.45					
1.5				0.45					
2.5				0.45					
3.4				0.45					
4.4				0.45					
5.4	10	4	9	0.45					
6.4	7	3	4	0.45					
7.4	9	3	6	0.45					
8.4	9	3	6	0.45					
9.4	8	4	9	0.45					
10.3	23	7	56	0.45		<1			0
11.3	23	8	79	0.45		<1			0
12.3	10	6	23	0.45					
13.3	15	5	27	0.45					
14.3	17	3	16	0.45					
15.3	9	5	18	0.45					
16.2	12	5	25	0.45					
17.2	10	5	21	0.45					
18.2	13	5	27	0.45					
19.2	14	5	31	0.45					
20.2	15	4	26	0.45					
21.2	13	4	23	0.45					
22.1	14	4	24	0.45					
23.1	13	4	23	0.45					
24.1	32	3	40	0.45					
25.1	29	3	36	0.45					
26.1	21	3	28	0.45					
27.1	19	3	25	0.45					
28.1	20	3	27	0.45					
29.0	15	3	21	0.45					
30.0	8	4	17	0.45					
31.0	9	5	23	0.45					
32.0	12	4	25	0.45					
33.0	10	5	28	0.45					
34.0	10	5	30	0.45					
34.9	10	5	28	0.45					
35.9	9	5	27	0.45					
36.9	10	5	29	0.45					
37.9	25	6	98	0.45					
38.9	19	6	74	0.45					
39.9	14	5	43	0.45					
40.8	20	8	133	0.45		<1			0
41.8	23	6	97	0.45					
42.8	24	7	122	0.45		<1			0
43.8	23	8	158	0.45		1.3			
44.8	23	7	117	0.45		<1			0
45.8	19	8	133	0.45		<1			0
46.8	22	7	117	0.45		<1			0
47.7	25	6	110	0.45					
48.7	26	7	137	0.45		<1			0
49.7	22	7	119	0.45		<1			0
50.7	22	7	122	0.45		<1			0
51.7	22	6	101	0.45					
52.7	18	4	49	0.45					
53.6	17	5	64	0.45					
54.6	18	7	104	0.45		<1			0
55.6	---	---	---	0.45					
56.6	---	---	---	0.45					
57.6	---	---	---	0.45					
58.6	---	---	---	0.45					
59.6	---	---	---	0.45					

Exploration No CPT-04 **Date Completed** Feb. 10, 2004
Comments:

Depth to Groundwater, feet 5

Horizontal Displacement	
Height of Nearest Slope Face, feet	0
Distance from Slope Face, feet	0
Ground Surface Grade, %	0%
Depth to Top of Layer of Concern	0
Depth to Bottom of Layer of Concern	1
Max. Displacement	0

Depth feet	**SPT** $N_{1\ 60}$	**SBT**	Q_{c1ncs}	**Response Acceleration, g**	**Vs** m/sec	**FS** Q_{c1ncs}	**FS Vs**	**Total Settlement**, in	**Horiz. Displace. Sloping Ground**, ft
0.5				0.45					
1.5				0.45					
2.5				0.45					
3.4				0.45					
4.4				0.45					
5.4	35	3	20	0.45					
6.4	28	3	18	0.45					
7.4	47	3	33	0.45					
8.4	22	3	16	0.45					
9.4	16	3	13	0.45					
10.3	22	3	18	0.45					
11.3	20	3	17	0.45					
12.3	14	3	12	0.45					
13.3	18	3	17	0.45					
14.3	12	3	11	0.45					
15.3	9	3	9	0.45					
16.2	7	4	11	0.45					
17.2	12	5	25	0.45					
18.2	10	4	16	0.45					
19.2	14	3	16	0.45					
20.2	13	3	15	0.45					
21.2	8	4	14	0.45					
22.1	7	5	17	0.45					
23.1	12	3	14	0.45					
24.1	11	3	13	0.45					
25.1	7	4	13	0.45					
26.1	5	5	13	0.45					
27.1	7	5	18	0.45					
28.1	21	3	28	0.45					
29.0	12	3	16	0.45					
30.0	10	3	13	0.45					
31.0	8	4	17	0.45					
32.0	13	3	18	0.45					
33.0	12	3	17	0.45					
34.0	11	4	24	0.45					
34.9	10	4	23	0.45					
35.9	10	4	22	0.45					
36.9	10	4	23	0.45					
37.9	13	4	30	0.45					
38.9	20	3	32	0.45					
39.9	12	4	28	0.45					
40.8	7	5	24	0.45					
41.8	8	5	26	0.45					
42.8	10	4	25	0.45					
43.8	14	3	23	0.45					
44.8	10	4	25	0.45					
45.8	10	4	24	0.45					
46.8	12	3	20	0.45					
47.7	12	3	21	0.45					
48.7	11	3	19	0.45					
49.7	9	3	16	0.45					
50.7	8	3	15	0.45					
51.7	11	3	21	0.45					
52.7	14	7	80	0.45		<1			0
53.6	19	5	70	0.45					
54.6	20	7	113	0.45		<1			0
55.6	18	6	85	0.45					
56.6	14	6	66	0.45					
57.6	15	6	73	0.45					
58.6	16	7	91	0.45		<1			0
59.5	14	6	71	0.45					

Exploration No CPT-05 **Date Completed** Feb. 10, 2004
Comments:

Depth to Groundwater, feet 5

Horizontal Displacement

Height of Nearest Slope Face, feet	0
Distance from Slope Face, feet	0
Ground Surface Grade, %	3%
Depth to Top of Layer of Concern	0
Depth to Bottom of Layer of Concern	1
Max. Displacement	0

Depth feet	**SPT** $N_{1\,60}$	**SBT**	Q_{c1ncs}	**Response Acceleration, g**	**Vs** m/sec	**FS** Q_{c1ncs}	**FS Vs**	**Total Settlement**, in	**Horiz. Displace. Sloping Ground**, ft
0.5				0.45					
1.5				0.45					
2.5				0.45					
3.4				0.45					
4.4				0.45					
5.4	13	3	7	0.45					
6.4	9	3	6	0.45					
7.4	20	3	14	0.45					
8.4	12	3	8	0.45					
9.4	9	3	7	0.45					
10.3	6	3	5	0.45					
11.3	6	3	5	0.45					
12.3	9	3	7	0.45					
13.3	8	3	8	0.45					
14.3	7	3	6	0.45					
15.3	7	3	7	0.45					
16.2	7	3	7	0.45					
17.2	6	3	6	0.45					
18.2	6	3	6	0.45					
19.2	9	3	10	0.45					
20.2	9	3	11	0.45					
21.2	9	3	10	0.45					
22.1	15	4	27	0.45					
23.1	23	5	57	0.45					
24.1	28	4	52	0.45					
25.1	28	5	72	0.45					
26.1	58	11	75	0.45					
27.1	71	11	92	0.45					
28.1	27	5	70	0.45					
29.0	18	5	49	0.45					
30.0	16	5	43	0.45					
31.0	16	4	32	0.45					
32.0	16	3	23	0.45					
33.0	12	4	26	0.45					
34.0	26	3	38	0.45					
34.9	30	3	44	0.45					
35.9	24	3	36	0.45					
36.9	11	5	34	0.45					
37.9	21	6	83	0.45					
38.9	32	3	51	0.45					
39.9	21	3	34	0.45					
40.8	20	3	32	0.45					
41.8	21	3	34	0.45					
42.8	22	3	37	0.45					
43.8	17	3	28	0.45					
44.8	16	3	28	0.45					
45.8	18	3	30	0.45					
46.8	11	4	29	0.45					
47.7	8	5	27	0.45					
48.7	14	4	37	0.45					
49.7	11	4	30	0.45					
50.7	19	3	34	0.45					
51.7	18	3	33	0.45					
52.7	10	4	28	0.45					
53.6	15	3	28	0.45					
54.6	15	3	28	0.45					
55.6	10	4	27	0.45					
56.6	10	4	27	0.45					
57.6	16	3	30	0.45					
58.6	16	3	31	0.45					
59.5	16	3	31	0.45					